EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS
Earthquake Engng. Struct. Dyn. **28**, 1361–1381 (1999)

VULNERABILITY OF REINFORCED CONCRETE FRAMES IN LOW SEISMIC REGION, WHEN DESIGNED ACCORDING TO BS 8110

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SUMMARY

The overstrength and ductility due to redistribution of internal forces are being investigated for three bay multi-storey reinforced concrete plane frames, using non-linear push-over analysis. These frames are designed to resist gravity loads, wind loads and a notional horizontal load in accordance with the British code BS 8110, which does not have any special provision for seismic loads. The results show that the overstrength factors for the three-, six- and ten-storey frames are respectively, 7·5, 5·6 and 2·2 times the design lateral loads, whereas, the ductility factors for the three frames are similar, and slightly greater than 2. These values yield a response modification factor of 18·0, 12·2 and 4·7 for the three-, six- and ten-storey frames, respectively. The effect of infill walls on the response modification factor is also being investigated, and a suitable response modification factor for assessing the vulnerability of reinforced concrete frames of about 10 storeys high is recommended. Copyright © 1999 John Wiley & Sons, Ltd.

KEY WORDS: push-over analysis; overstrength; ductility; response modification factor

INTRODUCTION

Buildings sited outside the seismic zones such as those in Singapore and Malaysia are not designed for seismic loads. These buildings are designed according to BS 8110, which does not have any seismic provisions.[1] However, due to far-field effects of earthquake in Sumatra, these buildings are occasionally subjected to tremors. Therefore in order to assess the vulnerability of these buildings to low seismic excitation, it is important to determine the overstrength and ductility of these buildings which are designed for wind loads and notional horizontal load. The possible sources of overstrength are: the higher strength of materials used in construction than that specified; larger structural members and reinforcement bar sizes than those required; use of conservative material models in design; load combinations; non-structural elements (e.g. infill walls) and structural elements (e.g, slabs) that are not included in estimating the lateral load

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CCC 0098–8847/99/111361–21$17·50

Received 21 April 1998
Revised 7 January 1999
Accepted 20 May 1999

capacity; increased resistance due to concrete confinement; minimum requirements prescribed by the design code on reinforcement and spacing of link; and redistribution of internal forces in the inelastic range.[2–5] The amount of overstrength contributed from many of the above sources is uncertain and cannot be relied upon in the design of the structure.[2] However, the overstrength due to redistribution of internal forces in the inelastic range, arising from simplification in design procedure, is dependable and can be estimated.

Bracci *et al*., have shown experimentally and analytically that the structures which are designed only for gravity loads, without seismic provisions, may still possess a substantial lateral strength capacity.[6] Based on the shake-table experimental test, and plastic and push-over analyses, the lateral strength capacity is found to be 15 per cent of total structural dead weight in a three-storey reinforced concrete frame model designed according to ACI 318-89 for gravity loads only.[7] Overstrengths have also been observed in structures designed with seismic provisions.[3,4,8–10] Bertero *et al*. noted that only the implicitly assumed overstrength in seismic codes could possibly cause a structure to survive in a higher seismic event than that designed.[8] Shahrooz and Moehle have observed a maximum base shear of 7·5 times the design value in an experimental study on moment-resisting reinforced concrete frame.[3] Mitchell and Paultre have carried out non-linear analyses on six-storey reinforced concrete frame structures and 12-storey reinforced concrete frame-wall structures, and found that (i) the overstrength factors of the frame structures designed for force level corresponding to ductility factors $\mu = 2{\cdot}0$ and $4{\cdot}0$ are 2·14 and 4·6, respectively, and (ii) the overstrength factors of the frame-wall structures with $\mu = 2{\cdot}0$ and $3{\cdot}5$ are 2·78 and 5·32, respectively.[4] These results have shown that structures designed with higher levels of ductility, and hence lower design force levels, can in fact exhibit higher lateral strengths than structures designed for lower levels of ductility. Uang showed that, for steel frames located in high seismic region, overstrength of a four-storey steel frame is about 40 per cent higher than that of a 12-storey steel frame.[9] Jain and Navin have assessed analytically the seismic overstrength of multistorey concrete frames designed for various seismic zones according to Indian codes and found that the average lateral strengths of the frames are from 2·84 to 12·7 times the unfactored design base shear, depending on the seismic zone.[10]

In seismic codes, the overstrength is incorporated through response modification factor, which is used to reduce the base shear induced in a structure when assumed to behave elastically under the worst credible event, one in 475 years earthquake. The amount of reduction depends on the overstrength and ductility of the structure, as illustrated in Figure 1. The design base shear V_d is defined as

$$V_d = \frac{V_e}{\Omega\mu} \tag{1}$$

where V_e is the elastic base shear; Ω the overstrength factor; and μ the ductility factor defined as

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{2}$$

where Δ_u is the ultimate displacement, and Δ_y the significant yield displacement. Thus, the response modification factor

$$\begin{aligned} R &= \Omega\mu \\ &= \text{overstrength factor} \times \text{ductility factor} \end{aligned} \tag{3}$$

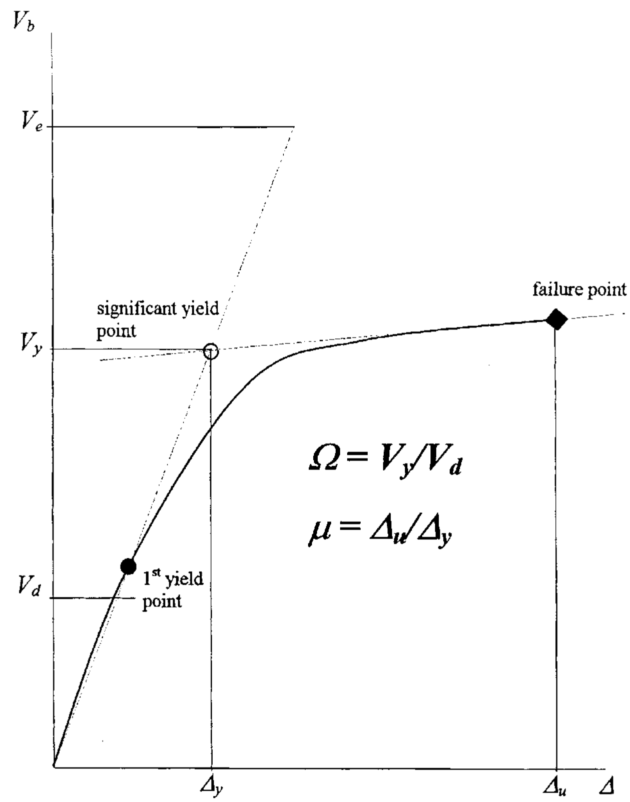


Figure 1. Frame overstrength factor and displacement ductility factor

In Canadian Code, the overstrength factor is accounted explicitly, whereas in the Uniform Building Code, it is accounted implicitly.[11,12]

This paper investigates the overstrength and ductility of reinforced concrete frames designed according to BS 8110.[1] The frames considered in this investigation are three-, six- and ten-storey three-bay, reinforced concrete plane frames. A non-linear push-over analysis which can predict the structural behaviour under lateral loads is used to calculate the base shear capacity and ductility of the concrete frames. The calculated overstrength and ductility are based on the redistribution of internal forces only.

OVERSTRENGTH AND DUCTILITY

Design of reinforced concrete frames

Typical three-, six- and ten-storey three-bay reinforced concrete symmetrical plane frames with bay width of 5500 mm and storey height of 3700 mm, have been designed according to BS 8110.[1] Sizes of beams and columns are identical throughout the height of the frame: 300×400 mm

beams and $300\,\mathrm{mm}^2$ columns for three-storey frame; 300×400 mm beams and 300×350 mm columns for six-storey frame; 300×400 mm beams and 300×400 mm columns for 10-storey frame. The slab adjacent to the support of a continuous beam acts as part of the beam section resisting the tensile forces. This would increase the flexural and shear strength of the beam section. However, it is difficult to estimate the effective width of the slab that would contribute to flexural tension in a beam. The values given in codes are generally on the conservative side. Hence, the contribution of the slab is not considered in this paper in order to estimate reliably the overstrength and ductility of the frame from the dependable source. For the design, dead load g_k of $4{\cdot}5\,\mathrm{kN/m^2}$, a live load q_k of $3{\cdot}5\,\mathrm{kN/m^2}$, a basic wind speed V of 32 m/s and a notional horizontal floor load of 1·5 per cent of characteristic dead weight of the floor, have been considered. A characteristic cube strength of concrete of $30\,\mathrm{N/mm^2}$ and a characteristic yield strength of steel of $460\,\mathrm{N/mm^2}$ have been used to calculate the section capacities.

The design of beams and columns are based on the critical moments, axial loads and shears obtained by considering the various load combinations required by BS 8110.[1] The combinations of the dead load, g_k, imposed load, q_k, and wind load, w_k, considered for the frame design are:

(i) $1{\cdot}4g_k + 1{\cdot}6q_k$

(ii) $1{\cdot}4g_k + 1{\cdot}4w_k$

(iii) $1{\cdot}2g_k + 1{\cdot}2q_k + 1{\cdot}2w_k$

(iv) $1{\cdot}0g_k + 1{\cdot}4w_k$

For robustness, BS 8110 requires the design ultimate wind load, $1{\cdot}4w_k$ or $1{\cdot}2w_k$, to be not less than the notional horizontal load of 1·5 per cent of the characteristic dead weight of the structure.[1] The wind load at the floor level of the frame, W_{ki}, are calculated, according to British Standard CP 3 as[13]

$$W_{ki} = C_f q_i A_i \tag{4}$$

where

$$q_i = 0{\cdot}613 V_s^2 \tag{5}$$

$$V_s = V S_1 S_2 S_3 \tag{6}$$

in which C_f is the force coefficient, taken as 1·1 in this study, A_i the frontal area at ith floor, V the basic wind speed defined as the 3-sec gust with 50 yr of return period, 10 m above ground in an open area with no obstructions, taken as 32 m/s in this study, S_1 the correction factor to account for variation in wind environment due to topography, taken as 1·0 in this study, S_2 the correction factor to account for general ground roughness, building dimensions and height above ground; the value corresponding to the height of the top of each part being determined from Table III of CP3,[13] and S_3 the correction factor to account for the statistical characteristics of the wind, taken as 1·0 in this study.

Figures 2(a), 2(b) and 2(c) show the design lateral load acting on the three-, six- and ten-storey reinforced concrete frames, respectively. The notional horizontal load is greater than the design ultimate wind load for the three-storey frame and close but greater than the design ultimate wind load for the six-storey frame. The effect of this lateral load is small compared to the effect of the design ultimate vertical live and dead loads. Thus, these two frames have been designed by considering only the vertical live and dead loads. For the 10-storey frame, the design ultimate

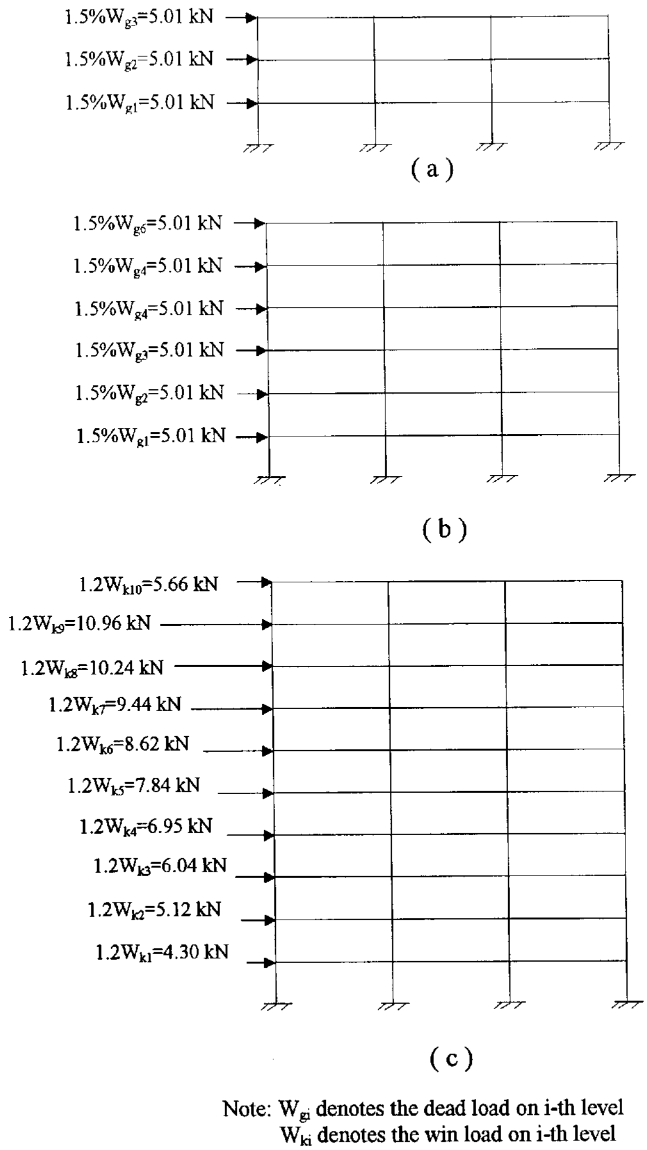


Figure 2. Design lateral loads considered for: (a) three-storey frame, (b) six-storey frame, and (b) ten-storey frame

wind load is greater than the notional horizontal load and has a significant effect; therefore, the combinations of wind and vertical loads, $1{\cdot}2g_k + 1{\cdot}2q_k + 1{\cdot}2w_k$, governed the design of this frame. BS 8110 allows the bending moment obtained from the elastic analysis of a continuous beam to be redistributed by up to 30 per cent as an indirect measure of ductility and to distribute bending moments away from the peak moment regions at the supports.[1] This reduces the congestion of reinforcement bars at the supports and makes the detailing and the construction easier. Moment

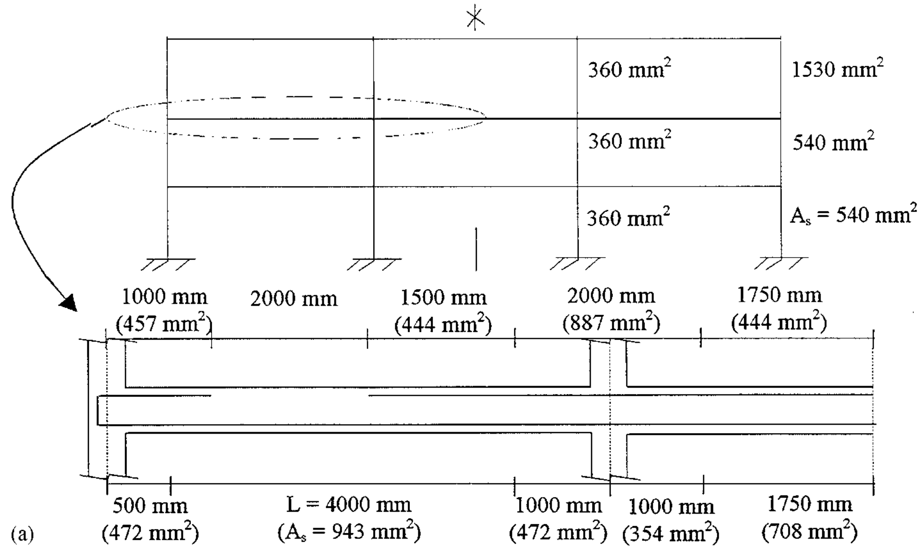
redistribution of 30 per cent has been carried out in the beams of three-storey frame, and moment redistribution of 10 per cent for the beams of six- and ten-storey frames. The reinforcing details of the frames are shown in Figures 3(a), 3(b) and 3(c); the transverse reinforcement for all the beams and columns consisted of 10 and 8 mm diameter bars, respectively, at R10-200 and R8-200, respectively.

Nonlinear push-over analysis

For structures subjected to lateral loads, a non-linear push-over analysis can be performed to obtain the structural strength capacities, and to locate plastic hinges and the regions potentially exposed to larger damage.[14] In such analysis, the magnitude of vertical loads is kept constant whereas the magnitude of lateral load, in a prescribed distribution, is progressively increased till a failure mode has occurred. The push-over analysis is terminated when one of the following occurs:

(i) The strain of concrete or reinforcing steel has exceeded the ultimate limit. In this study, the ultimate strain of confined concrete and steel is taken as 0·01 and 0·1, respectively.
(ii) A collapse mechanism caused by the formation of plastic hinges.
(iii) A reduction of 20 per cent of the base shear capacity of the frame has occurred. This is termed as excessive reduction in base shear capacity in this paper.
(iv) The inter-storey drift has exceeded a limit of 2 per cent. This is termed as excessive drift.
(v) The shear capacity of beam or column section has been exceeded. This is termed as local shear failure in this paper.

In this study, the non-linear push-over analysis has been performed using a finite element program system ABAQUS to calculate the lateral strength capacities of the three frames, taking into account the P–Δ effect.[15] The microscopic finite element approach, which enables a better understanding of the mechanics of concrete structures has been used in the non-linear push-over analysis. A one-dimensional quadratic Timoshenko beam element with six degrees of freedom has been used for modelling the beams and columns in the frames. The cross-sections of the beams and columns are divided into 20 layers of concrete. Each cross-section of the element is integrated numerically in order to trace the non-linear response accurately. The steel reinforcing bars are modelled as one-dimensional strain theory elements (rods) and superimposed on the mesh of beam elements used to model the plain concrete. A layer model for the beam element is shown in Figure 4. This modelling approach allows the concrete behaviour to be considered independently of the steel reinforcing bar. Effects associated with the steel reinforcing bar/concrete interface, such as bond-slip and dowel action, are modelled approximately by introducing tension stiffening to simulate load transfer across cracks through the steel reinforcing bar. The concrete stress–strain model described by Mander *et al.* shown in Figures 5(a) and 5(b), is adopted in the analysis.[16] Tension stiffening is approximated by concrete model in tension with a linear softening. The steel stress–strain model is assumed elastic–plastic with a 5 per cent of hardening ratio, as shown in Figure 5(c). The modelling adopted here is verified with many numerical and experimental models reported by other researchers, prior to the application to the problem considered herein.[6,17–19] The structural behaviours predicted by the finite element analysis are found to be in good agreement with the published analytical and experimental results.



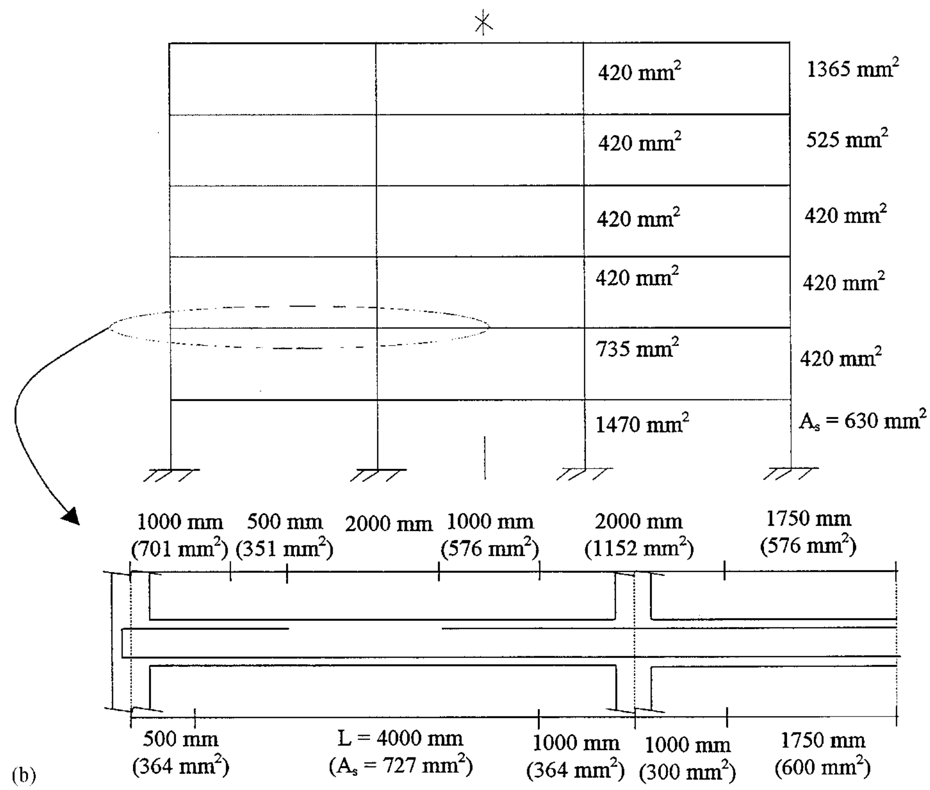
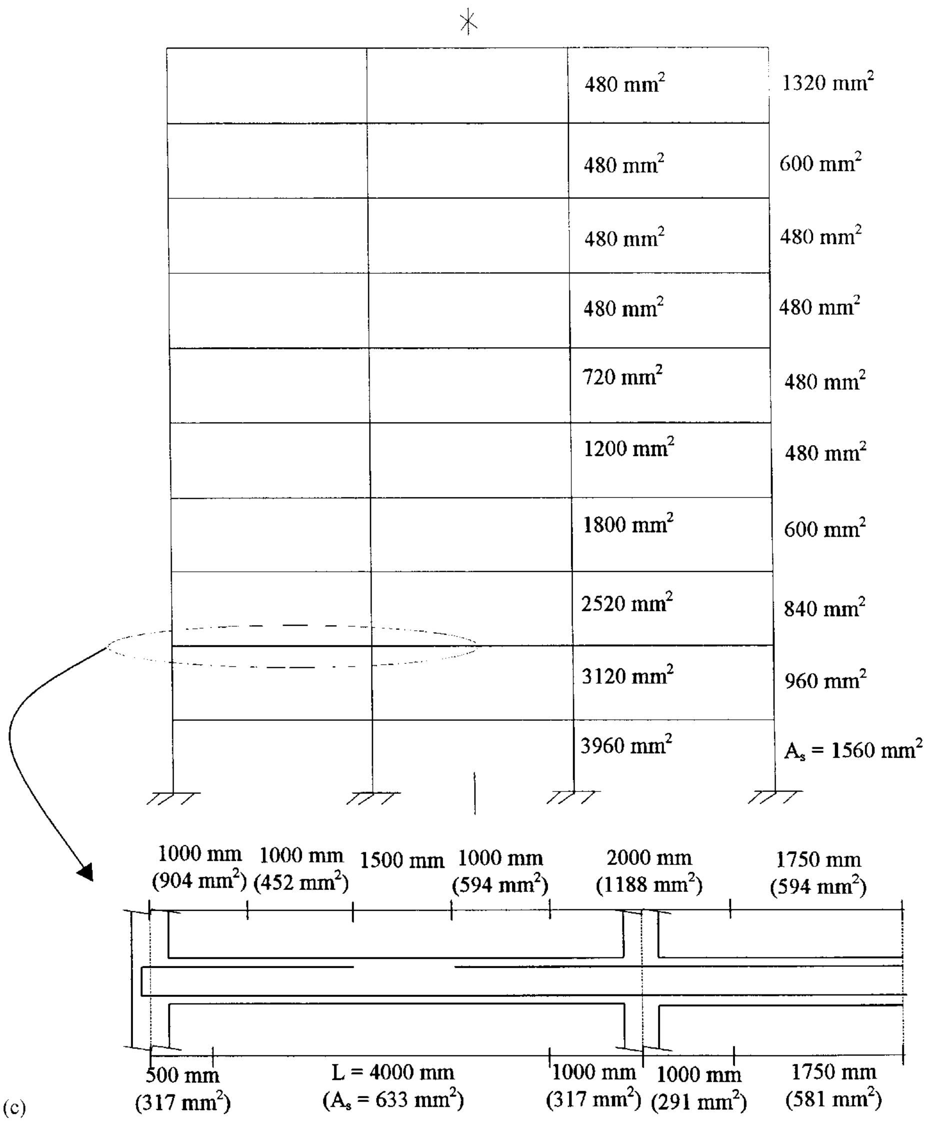


Figure 3. (a) Column and beam reinforcement for three-storey frame; (b) column and beam reinforcement for six-storey frame



(c)

Fig. 3(c). Column and beam reinforcement for ten-storey frame

Seismic loads

Short period building is normally considered to respond in the linear first mode shape. Thus, a linear distribution of static lateral loads is often used for seismic analysis of short buildings. For tall buildings, the contribution of higher modes becomes important and hence a non-linear distribution of static lateral loads is adopted to account for the influence of higher modes which

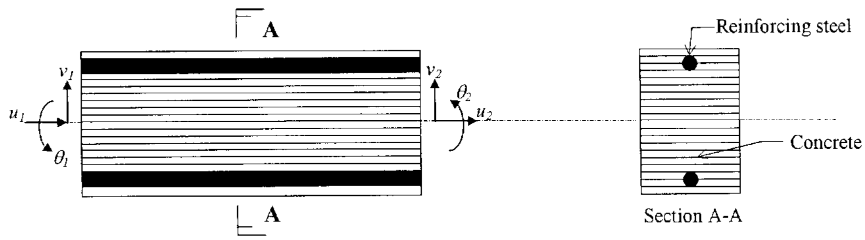


Figure 4. Layer model for the beam element

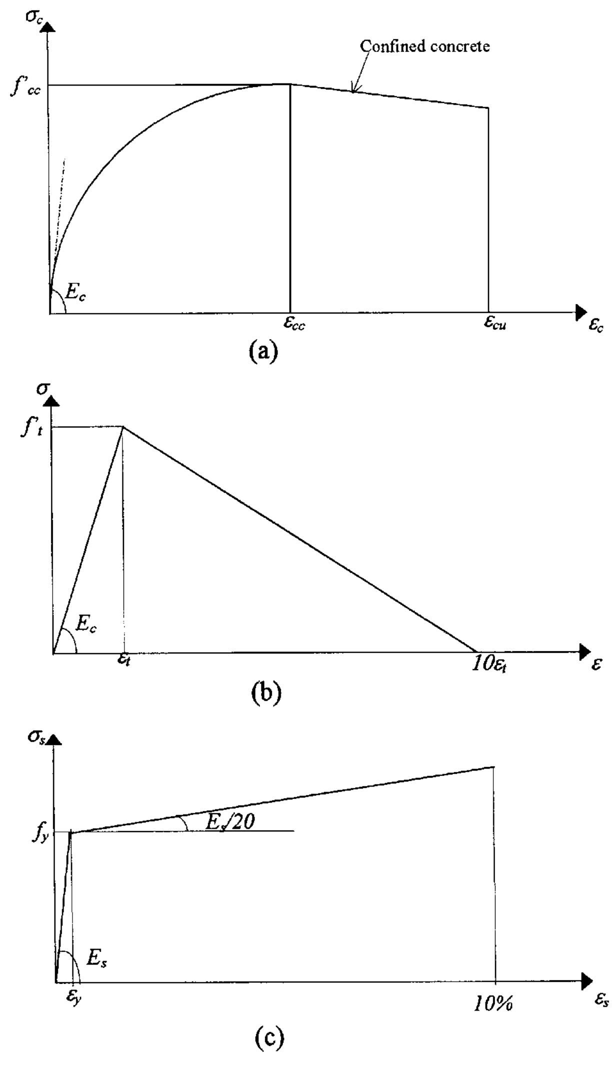


Figure 5. Constitutive relations for materials: (a) confined concrete in compression, (b) concrete in tension, and (c) reinforcing steel

increases the moments and shears in upper-storey members of long-period buildings. In this study, the non-linear distribution of static lateral loads recommended by Australian Standard has been adopted:[20]

$$F_x = \frac{G_{gx} h_x^k}{\sum_{i=1}^{n} G_{gi} h_i^k} V_b \tag{7}$$

where k is an exponent related to structure period, T; $k = 1{\cdot}0$; $T \leqslant 0{\cdot}5$ sec; $k = 1{\cdot}0 + 0{\cdot}5(T - 0{\cdot}5)$; $0{\cdot}5 \leqslant T \leqslant 2{\cdot}5$ sec; $k = 2{\cdot}0$; $T \geqslant 2{\cdot}5$ sec; $T \approx 0{\cdot}1n$; G_{gx} the total of gravity load located at storey x; G_{gi} the total of gravity load located at storey i; h_x the height above the base of the structure to storey x; h_i the height above the base of the structure to storey i; n the total number of storeys in structure and V_b the total horizontal earthquake base shear force.

The natural periods of the frames obtained through dynamic analysis, considering only the self-weight, were found to be 0·37, 0·66 and 1·03 sec. When the floor weights changed from the most common load combination ($1{\cdot}0g_k + 0{\cdot}4q_k$) to the ultimate load combination ($1{\cdot}2g_k + 1{\cdot}2q_k$), the natural periods obtained from dynamic analysis for the three-, six- and ten-storey frames changed from 0·40 to 0·44 sec, 0·74 to 0·82 sec and 1·18 to 1·34 sec, respectively. These values are reasonable as they are quite close to the natural periods of $0{\cdot}08n$–$0{\cdot}13n$ for the concrete frame obtained from Paulay and Priestley.[21] Since the expression for the period given in the code, lies within this range, this expression is used to compute the height-wise distribution of lateral load according to equation (7). Subsequent analysis revealed that any slight variation in the distribution hardly affected the ultimate strength and ductility of the frames considered in this study.

Overstrength

Figures 6(a) to 6(c) show the base shear versus top displacement curves of three-, six- and ten-storey frames obtained through push-over analysis, plotted in non-dimensional form. The failure modes of the frames are indicated in the figures for three different combinations of dead and imposed vertical load, that is, $1{\cdot}0g_k + 0{\cdot}4q_k$ (most common state), $1{\cdot}0g_k + 1{\cdot}0q_k$ (serviceability state) and $1{\cdot}2g_k + 1{\cdot}2q_k$ (ultimate state). It is seen that all the three frames are stiffer under most common vertical load combination ($1{\cdot}0g_k + 0{\cdot}4q_k$) than under the ultimate load combination ($1{\cdot}2g_k + 1{\cdot}2q_k$). The base shear strength and stiffness of the frames decrease as the vertical load increases.

The failure mode of three-storey frame changes from collapse mechanism for the most common vertical load combination ($1.0g_k + 0{\cdot}4q_k$) to local shear failure for the ultimate vertical load combination ($1{\cdot}2g_k + 1{\cdot}2q_k$), whereas for six- and ten-storey frames the failure mode changes from excessive inter-storey drift to local shear failure. As such, for all three frames the ultimate top-floor displacements are quite similar under most common and serviceability vertical load combinations. However, the ultimate displacements are smaller when subjected to the ultimate vertical load combination, due to premature shear failure.

The base shear capacities, V_b, of the three-, six- and ten-storey frames are summarized in Table I for different vertical load combinations considered. For the three- and six-storey frames, the base shear capacities under most common vertical load combination, as a ratio of 1·5 per cent W_g (base shear due to notional horizontal load) are 7·54 and 5·55, respectively. Although the notional horizontal load has been considered, the design of these frames was still governed by the gravity load only. Under the most common vertical load combination, the ten-storey frame, for

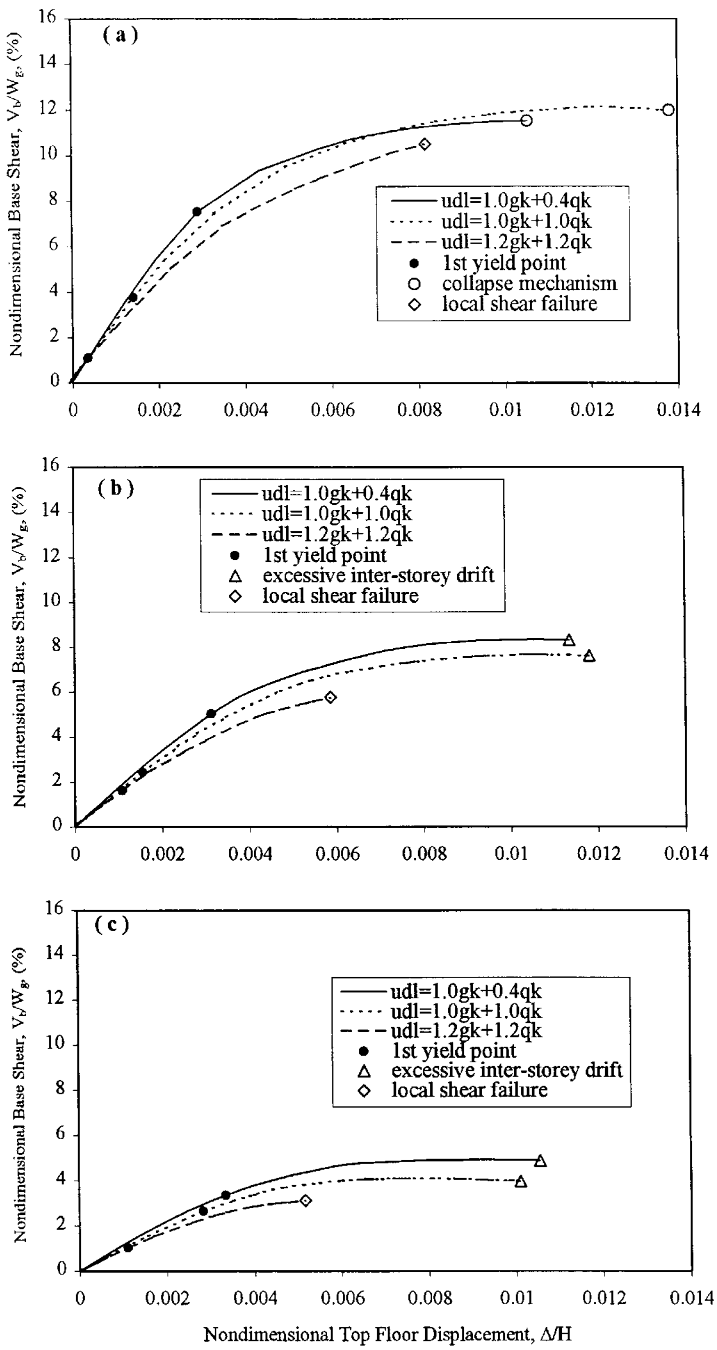


Figure 6. Base shear-top floor lateral displacement curve under various vertical load combinations for: (a) three-storey frame, (b) six-storey frame, and (c) ten-storey frame

Table I. Base shear capacities and ductility factors of three-, six- and ten-storey frames

Number of storeys	*udl* (kN/m)	V_b (kN)	$V_b/0{\cdot}015\ W_g$	$V_b/1{\cdot}2\ W_k$	$V_b/0{\cdot}01\ W_g$	Δ_u (mm)	μ	Failure mode
3	$1{\cdot}0g_k + 0{\cdot}4q_k = 26{\cdot}55$	113·38	7·54	—	11·31	116·80	2·39	Collapse mechanism
	$1{\cdot}0g_k + 1{\cdot}0q_k = 36{\cdot}00$	120·18	7·99	—	11·99	153·30	2·94	Collapse mechanism
	$1{\cdot}2g_k + 1{\cdot}2q_k = 43{\cdot}20$	80·51	5·35	—	8·03	90·64	2·48	Local shear failure
6	$1{\cdot}0g_k + 0{\cdot}4q_k = 26{\cdot}55$	166·86	5·55	—	8·32	251·90	2·19	Excessive inter-storey drift
	$1{\cdot}0g_k + 1{\cdot}0q_k = 36{\cdot}00$	152·88	5·08	—	7·63	262·00	2·38	Excessive inter-storey drift
	$1{\cdot}2g_k + 1{\cdot}2q_k = 43{\cdot}20$	92·81	3·09	—	4·63	130·30	1·83	Local shear failure
10	$1{\cdot}0g_k + 0{\cdot}4q_k = 26{\cdot}55$	162·66	—	2·16	4·87	390·40	2·16	Excessive inter-storey drift
	$1{\cdot}0g_k + 1{\cdot}0q_k = 36{\cdot}00$	134·86	—	1·79	4·04	373·60	2·32	Excessive inter-storey drift
	$1{\cdot}2g_k + 1{\cdot}2q_k = 43{\cdot}20$	92·14	—	1·23	2·76	191·30	1·71	Local shear failure

Note: $W_g = g_k \times 16{\cdot}5 \times$ number of storeys

which the design was governed by gravity and wind loads, has a base shear capacity of about 2·16 times the ultimate wind load ($1{\cdot}2W_k$). This is equivalent to 4·87 per cent of seismic gravity load, W_g, taken to be 100 per cent of dead load. The corresponding values for three- and six- storey frames are 11·31 and 8·32 per cent, respectively. Thus, the overstrength decreases as the number of storeys increases. Furthermore, as seen from this table, the overstrength decreases as the magnitude of the vertical load increases.

Ductility

The ductility factors of the three-, six- and ten-storey frames obtained from the push-over analysis are also given in Table I. For the three-storey frame, the ductility factors are quite similar under all the vertical load combinations, while the values vary little from the most common load combination state ($1{\cdot}0g_k + 0{\cdot}4q_k$) to the serviceability load combination state ($1{\cdot}0g_k + 1{\cdot}0q_k$) for the six- and ten-storey frames. The ductility values drop about 20 per cent when these two frames are subjected to the ultimate vertical load combination ($1{\cdot}2g_k + 1{\cdot}2q_k$). It is interesting to note that for each type of loading, the ductility is almost the same for both six- and ten-storey frames. As shown in Table I, the ductility of frames that failed in local shear is similar to that of frames failed in a collapse mechanism, but is less than that of frames failed in excessive inter-storey drift.

Table II. Response modification factor, R, for the three frames under most common vertical load combination ($1{\cdot}0g_k + 0{\cdot}4q_k$)

Number of storeys	Overstrength factor (from Table I)* Ω	Ductility factor μ	Response modification factor $R = \Omega\mu$
3	7·54	2·39	18·02
6	5·55	2·19	12·15
10	2·16	2·16	4·67

*$V_b/0{\cdot}015W_g$ or $V_b/1{\cdot}2W_k$

Response modification factor

Table II shows the response modification factor, computed according to equation (3), for the three frames under most common vertical load combination ($1{\cdot}0g_k + 0{\cdot}4q_k$). Due to redistribution of internal forces in the inelastic range, the response modification factors of the three-, six- and ten-storey frames are 18·02, 12·15 and 4·67, respectively. The larger values of the three- and six-storey frames are due to the fact that the designs of these frames are governed by gravity load and not by lateral load.

Effects of infill walls

Reinforced concrete frames are commonly filled with brick or concrete block masonry walls to meet the architectural and functional requirements. The infill walls which act compositely with the frame under loading can greatly increase the stiffness, but may reduce the overall strength and ductility due to premature failure caused by change of failure mechanism. In order to determine the effects of infill walls, the frames shown in Figure 7 is investigated. This configuration is commonly adopted in Singapore in order to provide void deck for social functions.

When an infilled frame is subjected to lateral loading, a large portion of the load is transmitted to the infills through the joints of the enclosed frame. The effects of the infills are similar to the action of diagonal struts bracing the frame. Thus, the infill walls are modelled as equivalent diagonal struts in this study, as shown in Figure 7. The formulas for calculating the equivalent strut areas are taken from Mainstone[22] which were derived to depend on the panel aspect ratio and the panel stiffness to frame stiffness ratio. The effective strut area, unit in square inches (1 in = 25·4 mm), is given as

$$A_i = w_e t \tag{8}$$

where

$$w_e = 0{\cdot}175(\lambda h)^{-0{\cdot}4} w' \tag{9}$$

$$\lambda = \sqrt[4]{\frac{E_i t \sin(2\theta)}{4E_f I_c h'}} \tag{10}$$

where E_i is the modulus of elasticity of the infill material, unit in ksi; E_f the modulus of elasticity of the frame material, unit in ksi; I_c the moment of inertia of column, unit in in.^{4} and t the thickness of infill, unit in in.

The symbols h, h', w' and θ are indicated in Figure 8.

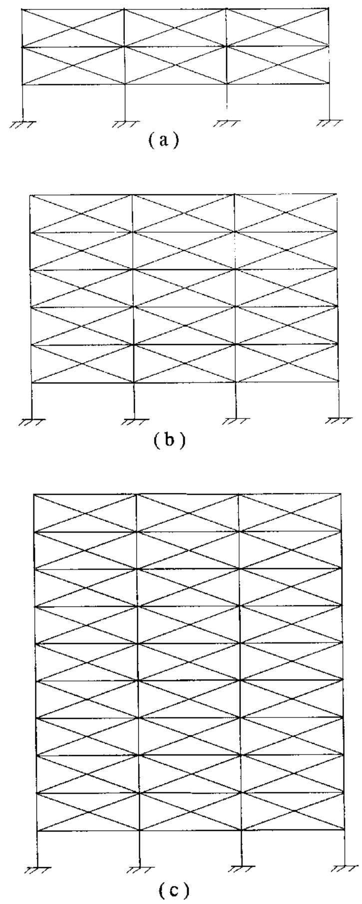


Figure 7. Modelling of infilled frames: (a) three-storey infilled frame, (b) six-storey infilled frame, and (c) ten-storey infilled frame

Figure 9 shows the stress–strain curve for concrete masonry given by Priestley and Elder.[23] This curve is adopted to model the material characteristic of the concrete masonry infill walls.

The base shear versus top displacement curves of three-, six- and ten-storey infilled frames under different combinations of vertical loads are shown in Figures 10(a), 10(b) and 10(c) and the base shear capacities are summarized in Table III. For the three- and six-storey frames, the base shear capacities under the most common vertical load combination ($1{\cdot}0g_k + 0{\cdot}4q_k$), as the ratios of 1·5 per cent W_g (base shear due to notional horizontal load) are 8·35 and 9·68, respectively. Under the most common vertical load combination, the 10-storey infilled frame for which both gravity and wind loads govern the design, has a base shear capacity of about 6·82 times the ultimate wind load ($1{\cdot}2W_k$). These values are greater than those of the corresponding bare frames. The ductility factors for three-, six- and ten-storey infilled frames are 1·89, 1·47 and 1·01, respectively, under the most common vertical load combination ($1{\cdot}0g_k + 0{\cdot}4q_k$). These values are smaller than those of the corresponding bare frames, particularly for six- and 10-storey frames,

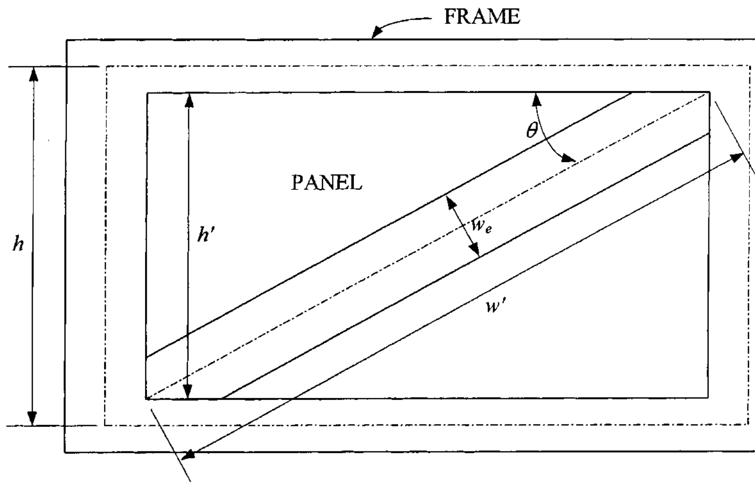
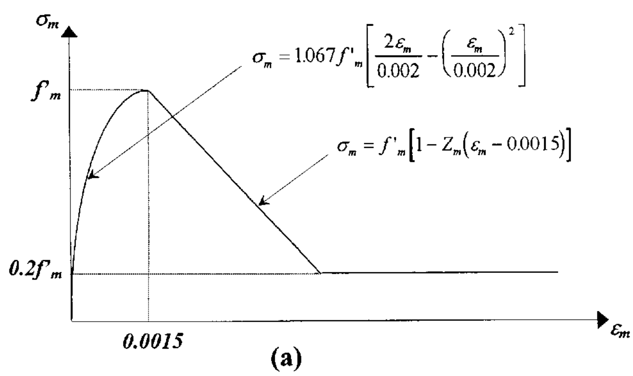


Figure 8. Dimensions of an idealized infilled frame as a frame-diagonal strut system



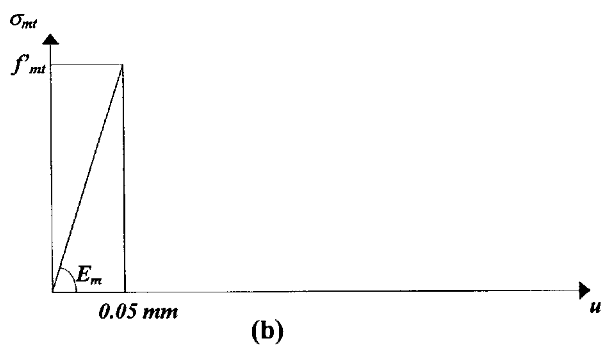


Figure 9. Constitutive relations for concrete masonry: (a) concrete masonry in compression, and (b) concrete masonry in tension

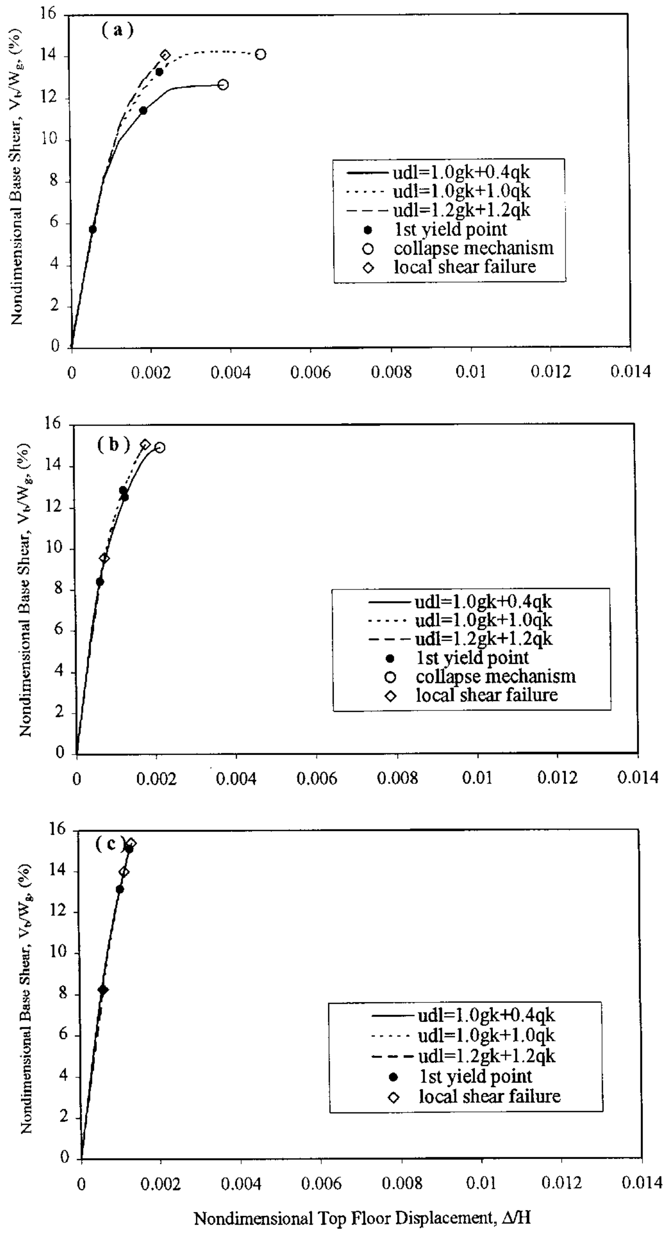


Figure 10. Base shear-top floor lateral displacement curve under various vertical load combinations for: (a) three-storey frame, (b) six-storey frame, and (c) ten-storey frame

Table III. Base shear capacities and ductility factors of three-, six- and ten-storey infilled frames

Number of storeys	*udl* (kN/m)	V_b (kN)	$V_b/0{\cdot}015\ W_g$	$V_b/1{\cdot}2\ W_k$	$V_b/0{\cdot}01\ W_g$	Δ_u (mm)	μ	Failure mode
3	$1{\cdot}0g_k + 0{\cdot}4q_k = 26{\cdot}55$	125·50	8·35	—	12·52	42·98	1·89	Collapse mechanism
	$1{\cdot}0g_k + 1{\cdot}0q_k = 36{\cdot}00$	141·44	9·41	—	14·11	53·42	1·96	Collapse mechanism
	$1{\cdot}2g_k + 1{\cdot}2q_k = 43{\cdot}20$	115·04	7·65	—	11·48	27·05	2·06	Local shear failure
6	$1{\cdot}0g_k + 0{\cdot}4q_k = 26{\cdot}55$	291·00	9·68	—	14·52	48·26	1·47	Collapse mechanism
	$1{\cdot}0g_k + 1{\cdot}0q_k = 36{\cdot}00$	274·90	9·14	—	13·71	39·90	1·35	Local shear failure
	$1{\cdot}2g_k + 1{\cdot}2q_k = 43{\cdot}20$	175·42	5·83	—	8·75	16·65	1·13	Local shear failure
10	$1{\cdot}0g_k + 0{\cdot}4q_k = 26{\cdot}55$	512·87	—	6·82	15·35	49·83	1·01	Local shear failure
	$1{\cdot}0g_k + 1{\cdot}0q_k = 36{\cdot}00$	451·96	—	6·01	13·53	42·55	1·06	Local shear failure
	$1{\cdot}2g_k + 1{\cdot}2q_k = 43{\cdot}20$	275·84	—	3·67	8·26	22·11	1·00	Local shear failure

Note: $W_g = g_k \times 16{\cdot}5 \times$ number of storeys

because of change of the failure mode. In the case of ten-storey infilled frame local shear failure in beam attributed to smaller ductility. The infill walls have increased the lateral strengths of three-, six- and ten-storey frames to 12·5, 14·5 and 15·4 per cent of the seismic gravity loads from 11·3, 8·3 and 4·9 per cent, respectively, as shown in Figures 11(a)–11(c).

The dominant period of earthquake accelerations near a fault (say 50–100 km) is generally low, and thus the infilled frames which are stiffer compared to bare frames will attract more forces, as shown in Figure 12 ($SA_1 > SA_2$). However, this is not necessarily true for buildings sited far away from a fault (say 300–500 km), where the weak long period components of shear waves are dominant, as shown in Figure 12 ($SA_3 < SA_4$). It is also evident from Figures 11(a)–11(c) that the infill wall increases the stiffness of the frame and as a result, the frequencies of the frames will be lowered. This causes an infilled frame sited far away from a fault (300–500 km) to attract less force, as the ground motion is dominated by long period shear waves. Hence, in the case of buildings in places like Singapore, the ratio of supplied strength to demand strength would be definitely higher for an infilled frame than a bare frame.

Table IV shows the response modification factors for the 10-storey frame, for which the design is governed by both lateral and vertical gravity loads. The infill walls have increased the response modification factor of the bare frame by more than 1·5 times. Although the ductility factor of the infilled frame is smaller than that of the bare frame caused by premature failure, the overstrength factor is greater. This yields a greater response modification factor for the infilled frame. The response modification factor reduces by about 50 per cent when the vertical load increases from the most common loading condition to the ultimate loading condition for both the bare and infilled frames.

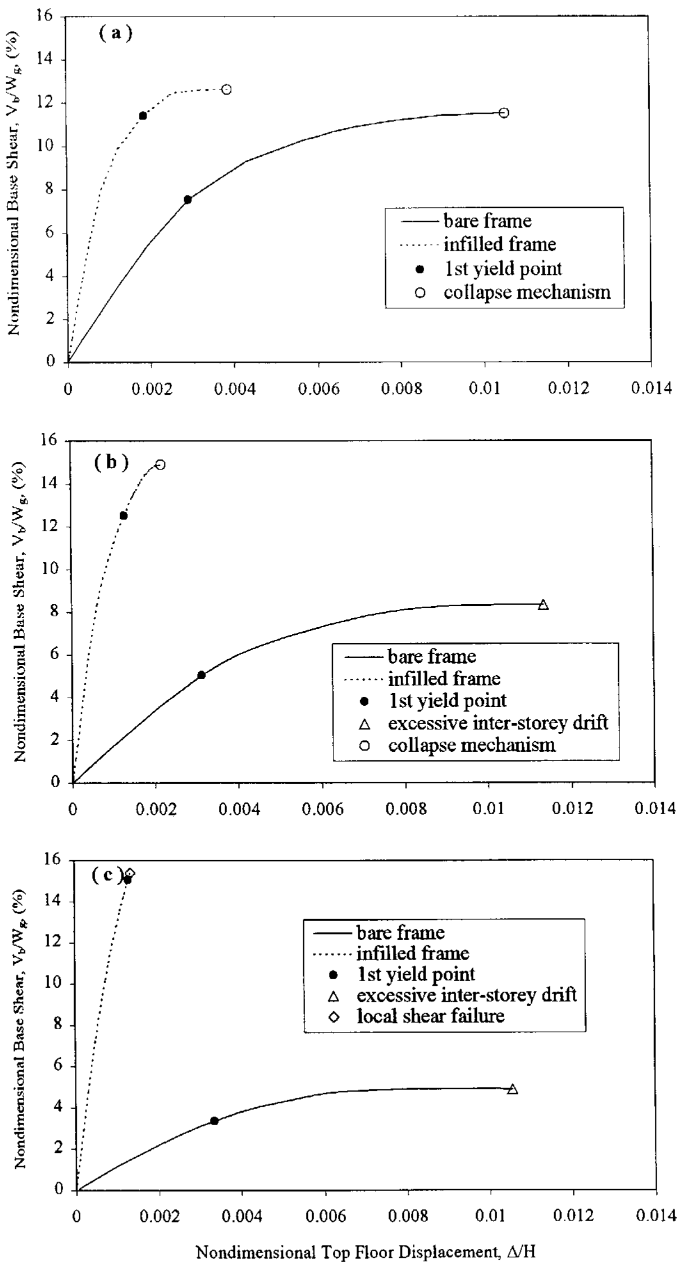


Figure 11. Base shear-top floor lateral displacement curve of bare and infilled frames under $1{\cdot}0g_k + 0{\cdot}4q_k$ for: (a) three-storey frame, (b) six-storey frame, and (c) ten-storey frame

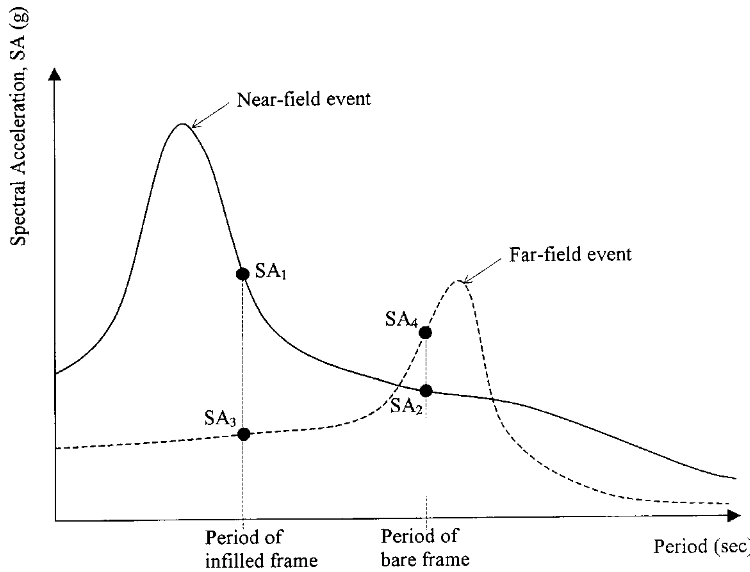


Figure 12. Schematic representation of spectral acceleration due to near- and far-field events, for the ten-storey bare and infilled frames

Table IV. Response modification factor, R, for the 10-storey frame with and without infills under various vertical load combinations

udl (kN/m)	Bare frame			Infilled frame		
	Overstrength factor (from Table I)* Ω	Ductility μ	Response modification factor $R = \Omega\mu$	Overstrength factor (from Table III)* Ω	Ductility μ	Response modification factor $R = \Omega\mu$
$1{\cdot}0g_k + 0{\cdot}4q_k = 26{\cdot}55$	2·16	2·16	4·67	6·82	1·01	6·89
$1{\cdot}0g_k + 1{\cdot}0q_k = 36{\cdot}00$	1·79	2·32	4·15	6·01	1·06	6·37
$1{\cdot}2g_k + 1{\cdot}2q_k = 43{\cdot}20$	1·23	1·71	2·10	3·67	1·00	3·67

$^*V_b/1{\cdot}2W_k$

CONCLUSION

The push-over analyses of reinforced concrete frames designed to BS 8110 reveal that due to the redistribution of internal forces in the inelastic range, the frames possess significant overstrength and considerable ductility when they are subjected to the most common loading condition ($1{\cdot}0g_k + 0{\cdot}4q_k$). In particular, for a 10-storey reinforced concrete frame, the response modification

factor which account for both overstrength and ductility is found to be about 4·7. Although the infill walls placed in all the upper storeys, except the first, could reduce the ductility significantly due to premature shear failure, the additional overstrength due to the presence of the infill walls increased the response modification factor to about 7·0. The response modification factors for bare and infilled frames under ultimate vertical loads ($1{\cdot}2g_k + 1{\cdot}2q_k$) are 2 and 4, respectively. Since this loading condition is very unlikely, in assessing the vulnerability of existing structures, the demand on base shear in reinforced concrete bare and infilled frames of about 10 storeys, could be computed using a response modification factor of 4·5, to err on the conservative side. The value of 4·5 is adopted based on the analytical results of the 10-storeys bare frame, as the response modification factor computed for the infilled frames could be less due to openings which are commonly found in real buildings. The previous study by Balendra *et al.* revealed that an earthquake of magnitude 7 Richter at 400 km away from Singapore, taken as the worst credible event, would produce an elastic base shear of 2, 3·3 and 4 per cent of weight when founded on stiff soil (fundamental period, $T_G = 0{\cdot}75$ sec), deep cohesionless soil ($T_G = 1{\cdot}20$ sec) and soft soil ($T_G = 2{\cdot}00$ sec).[24] By interpolating the analytical result obtained by Balendra *et al.*,[24] the elastic base shear for soil layer with fundamental period of 1·18 sec (same as the period of ten-storey bare frame under most common loading condition obtained through dynamic analysis) is found to be about 3·2 per cent of building weight, whereas for soil layer with fundamental period same as that of 10-storey infilled frame (0·53 sec), the elastic base shear is about 0·6 per cent . Thus, assuming a load factor of 1·2, the ultimate demand on base shear for the worst case would be $0{\cdot}032 \times 1{\cdot}2/4{\cdot}5 = 0{\cdot}85$ per cent of building weight, which is less than the notional horizontal load. In practice, the unaccountable factors which attribute to additional overstrength will further reduce the demand on base shear.

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